Strength Criteria of Infilled Rock Joints Tested in Ko Condition Using Triaxial Apparatus

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ABSTRACT

While determining rock strength to analyse stability of rock slopes, foundations on rock and design of underground openings, it is most important to account for the presence of discontinuities, namely joints, faults, shear zones, thrust zones, bedding planes etc. The stability depends on the geometry of discontinuities and the slope and orientation of excavated face. The most important factor is the shear strength of potential failure planes. The characterization of a discontinuity or a shear zone is not possible merely by looking at a specimen or by subjecting to conventional laboratory testing. The genesis of shear zone, its stress –strain history, the strength –deformation relationship, all combined to modulate its behaviour particularly when it approaches the state of limit equilibrium. The evaluation of strength criteria in field situation is more meaningful and therefore, Ko consolidated undrained tests were planned to conduct in triaxial. Based on extensive experimental results, it was found that the deviator stress, which controls the shear failure, is a better criterion for evaluating stress. As such, on the basis of Ko consolidated undrained (Ko C-U) triaxial tests, authors suggested strength criteria for discontinuous rock masses filled with gouge, which are found different than unconsolidated undrained (U-U) tests. The correlations to incorporate correction factor for thickness (t/a) for predicating shear strength is suggested. The correlation for predicting the shear modulus (G/G_0) is also presented.

Keywords: Discontinuities, Undulating and planar joints, Ko condition, Shear strength, Shear modulus, Transcendental equation

1. INTRODUCTION

All practical problems of slope instability are complex with uncertainty that usually shrouds the engineering behaviour of joints and other discontinuities, shear zones and slip surfaces constituting slide boundaries. The most important factor is the shear strength of the potential failure planes. The strength and deformational behaviour govern rock mass failure mechanism when it approaches the state of limit equilibrium (Barton et al.,1974; Sinha and Singh, 1999).

The presence of discontinuities divides the rock mass into blocks. In addition to the stress conditions, the geometry of the discontinuity surface (undulating/planar) also plays an important role in influencing the engineering behaviour of the rock mass (Lama, 1978; Sinha and Singh, 1999; 2000).

Patton (1966), Goodman (1970), Barton (1974), Ladanyi and Archambaul (1977), Barton and Chaubey (1977), Bandis et al., (1980), Hoek (1983) and many other researchers have discussed the shear strength of clean joints and proposed several models and empirical equations to evaluate shear parameters for the design of structures in and on rock mass.

It is reported that the choice of the correct shear parameters is difficult in the case of joints in relatively hard rock filled with weak and loose material varying from coarse gouge to sand to clay and constituting either shear debris and highly weathered products of rock material or deposited erosion products (Lama 1978, Lamas and Vutukuri 1978, Kutter and Rautenberg, 1979; Hassani and Scoble, 1985; Papaliangas et al., 1993; Sinha, 1993; Hoek, 1983; Nelsen, 1985).

The uncemented filling materials often termed as gouge may be in the form of partially loose to completely loose cohesive and non- cohesive weathered material and is deposited in open joints, shear joints, faults etc. The thickness of the fill material may very from a fraction of a micron to several millimeters. In case of tectonically crushed rocks the thickness of filling or gouge may increase up to several meters. Any clayey gouge in a sloped discontinuity makes the rock mass more prone to instability. When such a gouge becomes wet, it promotes sliding of the rock blocks (Hoek and Bray, 1981; Jumikis, 1983; Sinha, 1993).

Barton (1974) has also made an extensive review of the shear strength of filled discontinuities in rock. It is reported that the shear strength of joints with thick layer of filling is almost similar to the strength of filler and evaluated according to the principle of geotechnical engineering. The strength and stiffness of infilled joints change gradually with the relative filler thickness and influence of the surface roughness exists even for thicker fills (Kutter and Rautenberg, 1979).

Various workers have directed investigations on artificially created joints in rocks. However, study of the gap of knowledge in the development of pore water pressure during shearing, the influence of gouge thickness on strength of joint with dip of discontinuity surface have not been taken up in such investigation (Sinha and Singh, 1999) including simulation of rock joint test in field condition, i.e. K_0 test in laboratory for minimizing differences.

As such the study was undertaken to observe the behaviour of clayey gouge material along discontinuity surface in rock mass. The materials retrieved from the actual discontinuities and the materials ambient to it was studied under Ko consolidated undrained (Ko C-U) condition at various strain rates **(**ε = 5-80mm/hr)**,** thickness of gouge (t = 5-30mm) and dip angles (β = 5^0-50^0) both for undulating and planar surface profile in laboratory using computer controlled triaxial testing system. Need was felt to extend Barton's work for the prediction and assessment of the shear strength of discontinuity surfaces of rock mass filled with clayey gouge simulated to field condition.

Based on extensive laboratory test results a new shear strength criterion of joints filled with gouge after modification of Barton's equation has been suggested for slope stability analysis, foundation problem and analysis for underground openings (Sinha 1998; Sinha and Singh, 2000) and is presented in the paper.

2. EXPERIMENTAL MATERIAL

2.1 Gouge Material

Samples of gouge material (Kaliasaur shale) were collected from Kaliasaur landslide site located on Rishikesh- Badrinath road at about 18km east of Srinagar (Garhwal Himalya). The samples collected from discontinuities (Figs. 1 and 2) were taken to study their properties and shear strength behaviour of infilled joints in Ko-consolidated underained condition for representing field condition in laboratory. The gouge samples collected were pulverized in mortar and pestle till all material passed through 425 micron of Indian Standard sieve without rejecting any residue retained on 425 micron sieve to maintain homogeneous mineralogical compositions as present in the field situation (Sinha, 1993;, Sinha and Singh, 1999).

2.2 Physical Properties

The liquid limit (W_L) and plasticity index (W_P) were found in order of 24-26% and 5-6% respectively. The silt and clay fraction determined by washing method were found 46.9% and this was further analysed through laser particle size analyzer (Malvern Type 3600E) and the clay fraction (less than 2 micron size) was found in order of 2-3%. The gouge material used for experimental work revealed clay of low plasticity index ie CL group (Sinha and Singh, 2000).

2.3 Chemical Analysis

The chemical compositions of samples collected from two different locations are - SiO_2 (57.08-59.55%); Al_2O_3 (15.04-27.26%); Na_2O (4.34-0.32%); MgO (4.09-2.16%); P_2O_5 (0.42-0.10%); K_2O (1.62-0.77%); CaO (7.85-0.01%); TiO (0.540.-66%); MnO (0.04-0.03%) and Fe_2O_3 (4.97-5.77%). The variation in chemical composition may be due to the leaching out of chemical constituents monsoon season (Sinha and Singh, 1999).

2.4 XRD and DTA

XRD and DTA were carried out. XRD analysis revealed the presence of kaolinite mineral as accessory component associated with bulk of sample. DTA indicated strong endothermic reactions in the temperature range 70-100^{0}C and 550-6000C due to loss of mechanical and chemical water respectively. It was followed with exothermic reactions in the temperature range of 900-940^{0}C due to the formation of high temperature phases showing the mica group of clay minerals. The presence of kaolinite group clay mineral as accessory was supported by XRD, DTA and chemical analysis (Sinha and Singh,1999).

3.0 SIMULATED ROCK JOINT

3.1 Undulating and Planar Type of Joints

38mm diameter cylindrical specimens of Perspex (plastic resin) have been prepared. Each specimen comprised of two parts with one end flat and the other end having undulating or planar joints (Figs. 3 & 4). Such Perspex specimens were prepared for five angles (β) of undulating and planar joint. These angles are 5^0, 20^0, 30^0, 45^0 and 50^0. The height of the cylindrical specimen has been kept as 76mm after filling the gouge material in the joints. The testing has been performed for four thicknesses (t) of gauge material - 5, 10, 20 and 30mm. The provision of a hole of 2mm diameter was also made to saturate the gouge material by applying back pressure and to measure the pore water pressure during shearing (Figs.3a & b). The set of simulated rock joints (β= 5^0, 20^0, 30^0, 45^0 and 50^0), both undulating and planar, were fabricated as per Goodman (1970) considering JRC = 20 & 0 respectively (Barton 1974).

4.0 TRIAXIAL TEST

4.1 Dry Weight of Gouge for Test Specimen

The volume of shear zone for various thicknesses (t = 5, 10, 20, and 30mm) of simulated dummy rock joints alongwith dry weight of gouge material were calculated for various thickness at dry density (γd =15,00kN/m^3) are given in Table1.

4.2 Preparation of Test Specimen

The weighed quantity of oven dried pulverized gouge material (Table 1) was taken and it was thoroughly mixed after adding 15% distilled water (Fig. 5). The test specimens were prepared maintaining desired thicknesses (t = 5, 10, 20, and 30mm) at dip angles (β = 5^0, 20^0, 30^0, 45^0 and 50^0). The typical rock joints filled with gouge both for undulating and planar are shown in Figs.6a & b.

4.3 Ko Consolidated Undrained Test

Before starting Ko consolidate undrained (Ko C-U) test, the confining pressure (σ_3 =100kPa) and back pressure (u_0 = 75kPa) were simultaneously applied and saturated by saturation ramp technique. There after, the test specimens were consolidated under Ko condition by applying σ_1 = 200kPa and 300kPa terminal effective axial stresses simulating to the field situation for studying the strength criterion using triaxial apparatus at various strain rates (ε' = 5, 20 and 80mm/hr for undulating joints and 5mm /hr in case of planar joints) (Menzies, 1988 and Sinha, 2002).

Table 1 - Volume and dry weight of gouge (Sinha, 1993)

Thickness of Gouge, t (mm)	Volume of Shear Zone (cc)	Dry weight of gouge (g) (in filled material)
5	5.67	8.51
10	11.34	17.02
20	22.68	34.04
30	34.02	51.06

On the basis of plots between effective confining pressure (σ'_3) and effective axial stress (σ'_1), the Ko values (σ'_3/ σ'_1) were evaluated both for undulated and planar type of joints filled with gouge. The range of Ko values as obtained at various dip angles (β= 5^0, 20^0, 30^0, 45^0 and 50^0) and strain rates (ε'= 5,20 and 80mm /hr and 5mm /hr) in case of undulating and planar joints respectively consolidated at terminal effective axial stress (σ'_1) between 200 and 300 kPa in Ko condition are tabulated in Table 2.

Table 2 - Ko value of infilled rock joints (Sinha, 1993)

Dip Angle (β)	Range of Ko Value Considering Different Thickness of Gouge Material	
	Undulating	Planar
	For 5 mm/hr	*For 5 mm/hr*
5^0	0.33-0.70	0.33-0.73
20^0	0.53-1.05	0.57-1.04
30^0	0.42-0.85	0.43-0.93
45^0	0.10-0.80	0.67-1.01
50^0	0.43-0.90	0.70-1.01
	For 20 mm/hr	
20^0	0.42-0.80	
30^0	0.33-0.73	
45^0	0.40-1.00	
50^0	0.40-0.80	
	For 80 mm/hr	
20^0	0.47-1.23	
30^0	0.43-0.73	
45^0	0.35-0.87	
50^0	0.45-0.67	
Average Lower and Upper Limits	0.39-0.87	0.54-0.94

5.0 STRESSES ON FAILURE PLANES

5.1 Failure Planes

Rock masses are discontinuous and their instability depends on the geometry of discontinuities and the slope and orientation of excavated face. The most important factor is the shear strength along the potential failure planes (Sinha, 1993). The magnitude of shear stress and effective normal stress on the failure plane can be obtained by using the following Equations (Sinha and Singh, 1999).

$$\tau = \frac{\sigma_1 - \sigma_3}{2} \sin 2\beta \tag{1}$$

$$\sigma'_n = \frac{\sigma'_1 + \sigma'_3}{2} + \frac{\sigma_1 - \sigma_3}{2} \mathrm{Cos} 2\beta \tag{2}$$

$$\phi'_j = \tan^{-1}\left(\frac{\tau}{\sigma'_n}\right) \tag{3}$$

where

σ_1 = axial stress,

σ_3 = cell pressure,

σ'_1 = effective axial stress

σ'_3 = effective cell pressure,

β = angle between joint plane and major principal plane (designated as dip angle)

τ = shear stress at failure plan,

σ'_n = effective normal stress at failure plane, and

ϕ'_j = effective angle of internal friction at failure plane (Fig.7).

5.2 Modified Failure Plane

Goodman (1970) and Goodman et al. (1972) demonstrated and examined the influence of the thickness of the filling material (Kaolinite clay) in granite and sandstone joints and reported that for very thickness of filling material there is augmentation of the strength as a virtue of the geometry of the rough walls of the joint in direct shear box. However, the presence of such a condition may create different failure patterns in triaxial test. Sinha (1993) and Sinha and Singh (1999) observed that the failure pattern of undulating (rough) joint filled with gouge was not following the plane inclined at the joint, i.e dip angle. The filled joint showed deformation and bulging with not well defined failure plane in the case of undulating surface and necessitated modification in dip angle (Fig. 8). The failure of joint filled with gouge in the case of planar profile, i.e smooth joint showed the failure exactly through the predetermined failure plane, i.e. dip angle (Fig. 9). With due consideration of the behaviour in triaxial condition and along with the modified dip angle β(m) the magnitude of shear stress and effective normal stress on modified failure plane can again be determined from the following relationships.

$$\tau_m = \frac{\sigma_1 - \sigma_3}{2} \sin 2\beta(m) \quad (4)$$

$$\sigma'_{n(m)} = \frac{\sigma'_1 + \sigma'_3}{2} + \frac{\sigma_1 - \sigma_3}{2} \mathrm{Cos} 2\beta(m) \quad (5)$$

$$\phi'_{j(m)} = \tan^{-1}\left[\frac{\tau_m}{\sigma'_{n(m)}}\right] \quad (6)$$

where

τ_m = modified shear stress,

$\sigma'_{n(m)}$ = modified normal stress,

β(m) = modified dip angle,

$$\beta(m) = \tan^{-1}\left[\frac{P+t}{D_f}\right] \quad (7)$$

D_f = final diameter, and

P & t = defined in Figs. 8 & 9.

6.0 RESULTS AND ANALYSIS

The failure mechanism of undulating and planar type of joints filled with gouge mainly depends on the surface profile of the discontinuity, thickness of the fill material (gouge), orientation of joint i.e. dip angle, drainage condition, test condition and type of equipment. Accordingly, on basis of experimental results carried out in Ko C-U condition at various thicknesses (t = 5, 10, 20 and 30mm) dip angle ($\beta = 5^0$, 20^0, 30^0, 45^0 and 50^0) strain rates (ε') at terminal effective stresses (σ'_1 = 200 and 300 kPa) were evaluated and plotted (Figs. 10 through16).

7.0 DISCUSSION

7.1 General

The systematic procedure of testing technique in Ko C-U condition of rock joint filled with gouge in triaxial has helped in studying the behaviour of shear strength of discontinuous rock mass as follows.

7.2 Effect of Thickness, Surface Profile, Dip Angle, Effective Axial Stress

Ko consolidated undrained tests were carried out to study the strength behaviour of rock joints filled with gouge simulating the field conditions. Lower and upper limits of Ko-values for various dip angles ($\beta = 5^0 - 50^0$), thicknesses (t = 5-30 mm) and strain rates (ε' = 5 to 80 mm/hr for undulating joints and at 5 mm/hr for planar joints) are tabulated in Table 2. The Ko value varies with the thickness of gouge material at same angle β and strain rate. The overall range of Ko increases with increase in dip angle for a one particular strain rate. However, it is suggested that the effect of dip angle may be neglected to evaluate Ko value. Hence averaging the Ko value, these were found in the range of 0.39 to 0.87 and 0.5 to 0.94 for undulating and planar joints respectively. Planar joints revealed higher Ko-value because the joint roughness showed influencing factor and reduced the squeezing tendency of the gouge.

7.3.1 Effect of Thickness and Strain-rate on Dip Angle

The plots between the differential axial stress and dip angles [Figs. 10(a & b) and 11(a & b)] at various thickness and strain rates revealed more or less similar behaviour carried out in unconsolidated undrained (U-U) condition (Sinha, 1993). The effect of strain rate was found up to dip angle of 30^0 and thereafter, this effect was not found significant. The lowest strength was observed between 30^0 and 48^0 irrespective of strain rates both for undulating and planar joints. This behaviour is in agreement with the work of other researchers (Donath, 1963; Bamford, 1969; Hoek, 1983, Ramamurthy, 1985; and Sinha and Singh, 1996). The experimental results indicate that the deviator stress controls the shear failure and is a better criterion to evaluate the shear strength for thick gouge (t/a>1.25).

7.4 Effect of Thickness on Pore Water Pressure Factor (r_u)

Figures 12 (a & b) and 13 (a & b) show the increasing trend of the pore water pressure factor (r_u) with the increase of thickness. As can be seen, slightly higher magnitude of (r_u) was found in major cases at lower effective axial stress both for undulating and planar joints. This behaviour is very useful for risk analysis in hilly seismic areas (Markland 1988).

7.5 Modification of Barton's Equation (1974)

7.5.1 Transcendental Equation

Based on the experimental data generated, the shear strength of discontinuous rock mass were calculated using Barton's equation, i.e.

$$\tau = \sigma'_n \tan[\mathrm{JRC}.\log_{10} \frac{\sigma_1 - \sigma_3}{\sigma'_n} + \phi'_b] \tag{8}$$

Both for modified and unmodified dip angles at various speed of shearing assuming JRC = 20 and $\phi'_b = 25.5^0$ for undulating joints filled with gouge and for planar joints JRC was assumed Zero (Fig. 3b). The calculated shear strength based on Barton's Eq. 8 was compared with the maximum shear stress [$(\sigma_1-\sigma_3)/2$] as obtained through triaxial tests. Figs. 14 (a to c) show the relationship between experimental and theoretical strengths and as can be seen , the points are almost close to the unity line excepting a slight upward shift from the unity line beyond 30^0 dip for undulating joints but for thick gouge it is close to unity line . The trend was not observed in case of planar joints filled with gouge and found very close to the unity line (Fig. 14c). The findings provided an insight into the derivation of the transcendental equation viz.,

$$\tan\phi'_j = \mathrm{Sin}\,2\beta.\tan\left[\mathrm{JRC}\,\log_{10}\frac{2\tan\phi_j}{\mathrm{Sin}\,2\beta} + \phi'_b\right] \tag{9}$$

where

ϕ'_j = frictional angle of joint filled with gouge,
β = dip angle (angle between joint plane and major principal plane),
JRC = joint roughness coefficients,
= 0 for smooth clean joint surface,
= 20 for rough or undulating surface,
τ = shear stress of fault or discontinuities,
$(\sigma_1 - \sigma_3)$ = deviator stress, and
ϕ'_b = basic frictional angle.

If $\sigma_3 = 0$, the equation will have the same form of Baton's equation of rough clean joint,

$$\tau = \sigma'_n \tan[\mathrm{JRC}.\log_{10}\frac{\mathrm{JCS}}{\sigma'_n} + \phi'_b] \tag{10}$$

where JCS is joint wall compressive strength.

The transcendental equation for planar joint may directly be derived by putting JRC=0 in Eq. 9 for predicting strength of joint filled with gouge,

$$\tan\phi'_j = \mathrm{Sin}2\beta.\tan\phi'_b \ (\text{condition } \tan\phi'_j = \tan\phi'_b) \tag{11}$$

When thickness will be large, i.e. t>>a, ϕ'_b is equal to sliding angle of friction along contact plane between the gouge and surface of the host rock (Perspex used as dummy rock for the study).

The transcendental equation derived for planar joint assuming JRC = 0 is proposed to predict the shear strength criteria. This type of situation will seldom be present in actual field conditions. However, it can be utilised in case of development of residual strength in post-failure condition similar to cut plane for low thickness encountered due to large movement.

7.5.1.1 New strength criteria

Accordingly the new strength criterion has been developed to modify the Barton's Eqs. 8 and 10 for the rock joints filled with gouge both for undulating and planar type of joints.

The modified equations are given below

$$\frac{\sigma_1 - \sigma_3}{2} = \sigma'_n . f . \tan\left[JRC \log_{10} \frac{\sigma_1 - \sigma_3}{\sigma`n} + \phi'_b \right] \qquad \text{Undulating joint} \qquad (12)$$

$$\frac{\sigma_1 - \sigma_3}{2} = \sigma'_n . f . \tan \phi'_b \qquad \text{Planar joint} \qquad (13)$$

where f is the correction factor defined in the following paragraphs and other parameters are defined earlier.

7.5.2 Correction factor

In the (Eq. 9) the factors such as joint roughness (JRC), orientation of joint (dip angle), asperities, pore water pressure and basic effective friction angle (ϕ'_b) are already covered excepting the function of thickness to amplitude ratio [f(t/a).] The following equations were used to evaluate the correction factor [f(t/a)].

$$f(t/a) = \left[\frac{\tan \phi'_j}{\text{Sin } 2\beta \tan\left(JRC \log_{10} \frac{2 \tan \phi_j}{\text{Sin} 2\beta} + \phi'_b \right)} \right] \qquad \text{Undulating joint} \qquad (14)$$

$$f(t) = \left[\frac{\tan \phi'_j}{\text{Sin } 2\beta . \tan\phi_b} \right] \qquad \text{Planar joint} \qquad (15)$$

On the basis of calculated mean values of f(t/a), both for undulated and planar joints filled with gouge, the relationship between f(t/a) and thickness to amplitude ratio (t/a) were plotted [Fig.15(a & b)]. The decreasing trend in f(t/a) was observed upto t/a = 10 by extrapolation and thereafter the decreasing trend was found which is insignificant. This is higher than suggested by Goodman (1974) and Papaliangas et al. (1993) after direct shear box. In triaxial this condition may be possible. The nature of curve may be considered exponential. The correlation $f(t/a) = x + ye^{-t/a}$ was considered where x and y are material constants. On solution the following correlations have been derived to apply correction factor (f) to the above modified equations (Eq. 9 through 13) are given below:

$$f(t/a) = 0.98 + 0.96\, e^{-t/a} \qquad - \qquad \text{Undulating joint} \qquad (16)$$

$$f(t) = 0.80 + 0.61 e^{-t/4} \qquad - \qquad \text{Planar joint} \qquad (17)$$

The effect of strain rates on shear strength did not show any substantial change, hence correction for strain rate was not incorporated.

7.5.3 Predication of shear modulus

The shear modulus was estimated from the plots between maximum shear stress and shear strain following initial tangent method. The normalized shear modulus (G/G_o)

considering G_o was the lowest shear modulus corresponding to thick gouge material (if t>>a) was also obtained. Since the attainment of failure condition does not seem to be possible in case of lower dip angle ($\beta<20^0$), hence shear modulus for dip angle 5^0 was not considered to develop correlation for the prediction of shear modulus. The plots between normalized shear modulus (G/G_o) and t/a ratio both for undulating and planar joints for Ko C-U test are shown in Fig. 16 (a & b).

As can be seen, the variation and scattering in results due to variation in dip angle and roughness of joints are evident. Hence, the average curve drawn was observed exponential and the equation $G/Go = a + be^{(-t/a)\ \tan\beta}$ was derived. The following correlations have been developed to predict shear modulus of discontinuities filled with gouge.

$$G/G_o = 1.46 + 7.13\, e^{(-t/a)\tan\beta} \quad - \quad \text{Undulating joint} \qquad (18)$$
$$G/G_o = 1.09 + 3.84\, e^{(-t/4)\tan\beta} \quad - \quad \text{Planar joint} \qquad (19)$$

8. CONCLUSIONS

- Based on extensive experimental results, the data generated provide insight into modifying the proposed Barton's (1974) equation for evaluating shear strength of rock joints filled with thick gouge (t/a>>1.25) tested in K_0 condition in laboratory and are given in Eqs. 12 & 13. The correction factors are dependable on different drainage conditions in triaxial (Sinha, 1993 and Sinha & Singh, 1999).
- The correlations derived to predict shear modulus along discontinuities of rock mass filled with gouge are given in Eqs. (18 and 19) both for undulating and planar joints.
- The magnitude of pore water pressure factor (r_n) may provide a tool to undertake risk analysis in seismic hilly region (Markland 1988).

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References

Bamford, W.E. (1969). Anisotropy and the natural variability of rock properties, Proc. Symp. Rock Mech. University, Sydney, pp.1-10.

Bandls, S.C. (1980). Experimental studies of scale effects on the shear strength and deformation of rock joints, Ph.D. Thesis, University of Leeds, Dept. of Earth Sciences,UK.

Barton, N. (1974). A review of the shear strength of filled discontinuities in rock. NGI Publication 105, Oslo, pp. 1-48.

Barton, N.R. and Chaubey, V. (1977). The shear strength of rock joints in theory and paractice, Rock Mech. 10, pp. 1-54.

Chappell, B.A. (1990). Rock mass characterization for dam foundation, J.of Geotech. Engg., ASCE, Vol. 116, No. 4, pp. 625-645.

Donath , F.A. (1963). Strength variations and deformational behaviour in anisotropic rock, Proc. Int. Conf. state of stress in the earth crust , Santa Monic Calfornia, pp. 280-297.

Goodman, R.E. (1970). The deformability of joints, determination of the insitu modulus of deformation of rock, Symp. Demer Colo., ASTM Spl. Tech. Paper 477, pp. 179-196.

Goodman, R.E. Heuge, F.E. and Ohnishi, Y. (1972). Research on strength deformability water pressure relationship for faults in direct shear, Reprint University of California.

Hoek, E (1983). Strength of jointed rock massess, Geotechnique 33, No.3, pp. 187-233.

Hoek, E and Bray, J.M.(1981). Rock slope engineering, Inst. of Minnig Metallurgy, London.

Jumikis, A.R. (1983). Rock Machanics ,2nd Edition, pp.411.

Kutter, H.K. and Rautenberg, A. (1979). The residual shear strength of filled joints on rock. Proc. 4th Symp, ISRM, Montreux, Vol. 1,pp. 221-227.

Ladanyi, B. and Archambaul, I.G. (1977). Shear strength and deformability of filled joints, Proc. Int. Symp. on Geotechnics of structurally complex formations, Italian geotech. Assoc., Capri., Vol. 1, pp 317-326.

Lama, R.D. (1978). Influence of thickness of fill on shear strength of rough rock joints at low normal stress, Grundlagan and Felsmechanik, Felsmechanik Kolloquim Karlsruhe, Trans. Tech. Publicaztions, Clausthal, pp. 56-66.

Lama, R.D. and Vutukuri, V.S. (1978). Handbook on mechanical properties of rocks. Vol. IV, Chapter 10, Trans. Tech. Publication.

Menzies, B.K. (1988). A computer controlld hydraulick triaxial testing system, Spl. Tech. Pub.977, Advanced triaxial testing of soil and rocks, ASTM Philadephia.

Nilsen, B. (1985). Shear strength of rock joints at low normal stresses – a key parameter for evaluating rock slope stability, Proc. Int. Symp on fundamentals of rock joints, Bjorkliden, 15-20 Sept., pp. 487-496.

Papaliangas, T., Hencher, S.R., Lumsden, A.C. and Manolopolou, S, (1997). The effect of frictional fill thickness on the shear strength of rock discontinuities Int. J. rock mech. min. sci. & geomech. abst.,Vol.30, No.2, pp. 81-91.

Patton, F.d. (1966). Multiple modes of shear failure in rock and related materials, Ph. D. Thesis, University of Illinois, USA, pp. 282.

Ramamurity T.(1985). Stability of rock massess,8th IGS Annual Lecture, IGC, Roorkee, pp.1-74.

Sinha, U.N. (1993). Behaviour of clayey gouge material along discontinuity surfaces in rock mass. Ph.D. Thesis, Dept. of Civil Engg. University of Roorkee, Roorkee (India).

Sinha, U.N. (1998). Influence of thickness of gouge in discontinuous Rock Mass, Proc. Golden Jubilee Conf.,IGC-98, 7-10 Dec., IIT, New Delhi, Vol.I, pp. 351-354.

Sinha, U.N. and Singh, B. (1999). Strength criteria for rock masses with gouge filled discontinuities, Jr. Rock Mechanics and Tunnelling Technology, Vol. 5, No. 2, Dec., pp. 109 – 136.

Sinha, U.N. and Singh, B. (2000). Testing of rock joints filled with gouge using a triaxial apparatus, Int. Journal of Rock Mechanics and Min. Sciences, U.K., Vol. 37, No. 6, pp. 963-992.

Turk, N. and Dearman, W.R., (1985). Investigation of some rock joint properties, roughness angle determination and joint closure. Proc. Int. Symp. on fundamentals of rock joints, Bjorkliden, 15-20 Sept., pp. 197-204.

Wyllie, D.C. (1999). Foundations on rock, 2nd edition E&FN Spon, Lord and New York, pp 48-89.

Fig. 1. Presence of gouge in rock joint

Fig. 2. Gouge material retrieved from joint

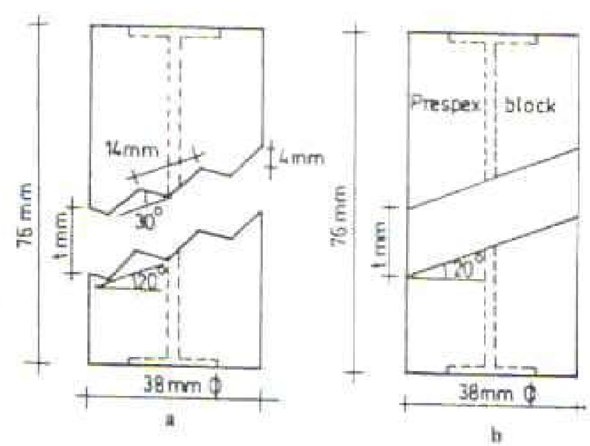

Fig. 2. Gouge material retrieved from joint

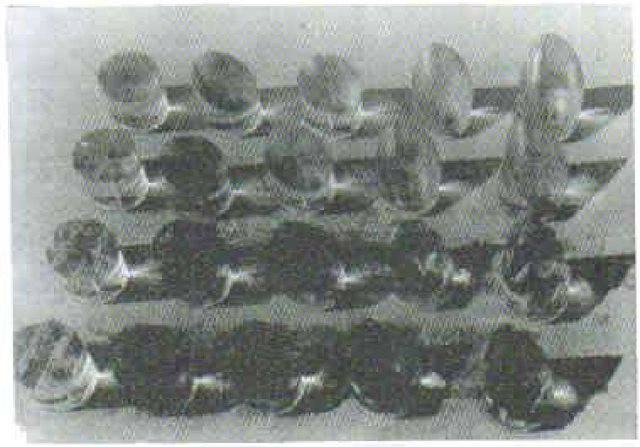

Fig. 4 Set of prefabricated undulating and planar joints (β=5°,20°,30°,45°, and 50°)

Fig. 5. Preparation of test specimen after proper mixing with water

Fig. 6. (a-b) Typical Test specimens of infilled joints (undulating and planar type of joints)

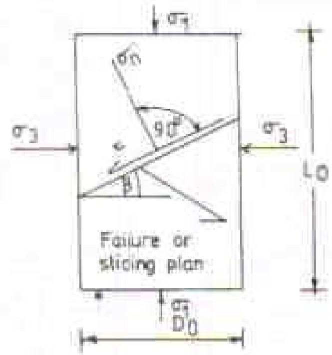

Fig. 7. Stress along failure plane

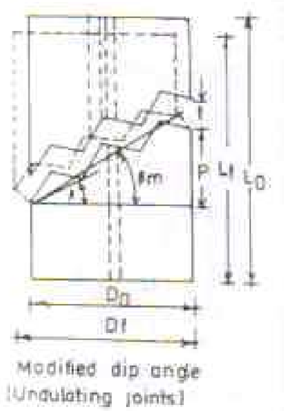

Fig. 8 Modification in dip angle for undulating joint

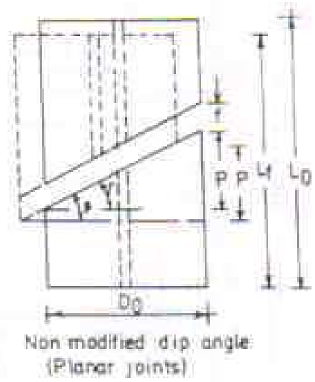

Fig. 9. Unmodification in dip angle for planar joint

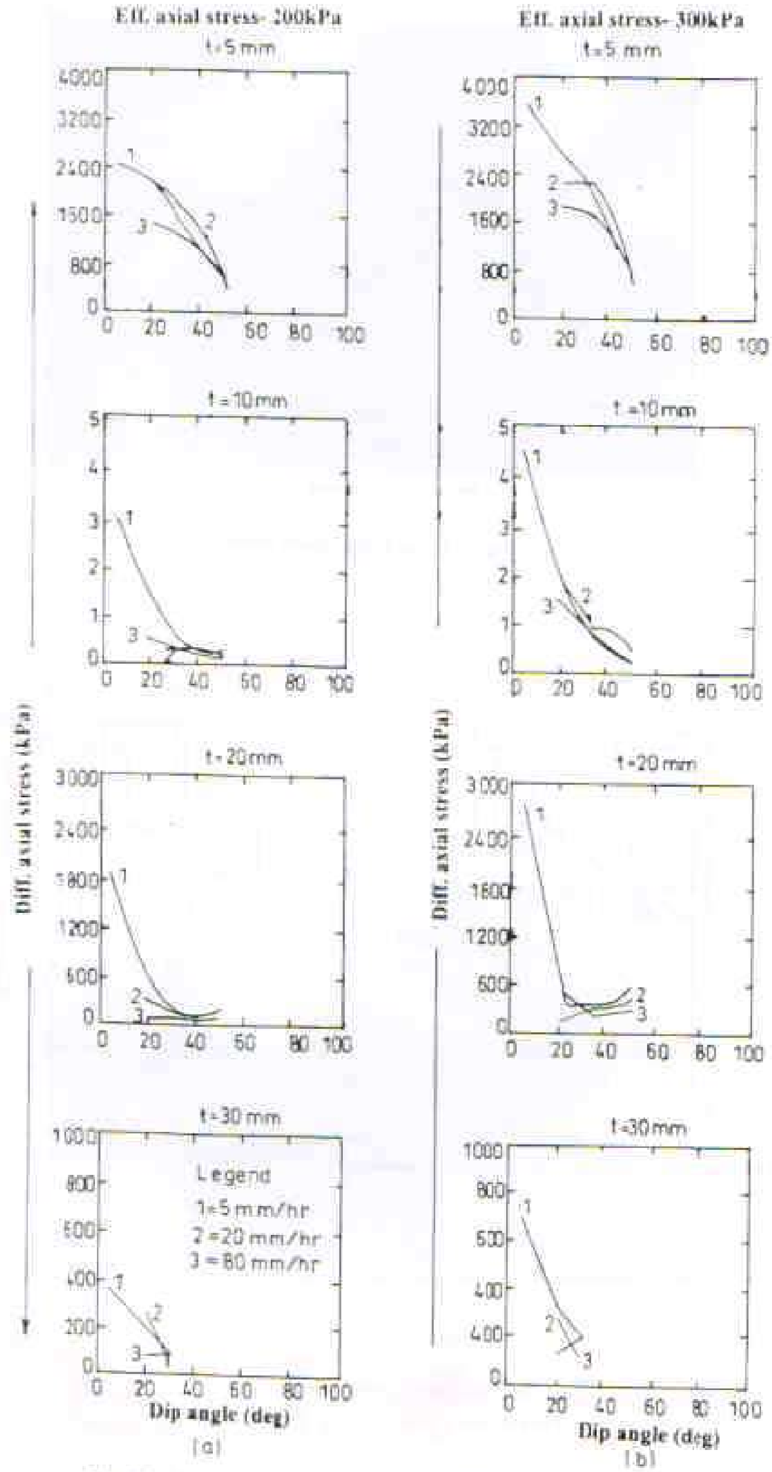

Fig. 10(a-b) Differential axial stress Vs Dip angle(Undulating joint)

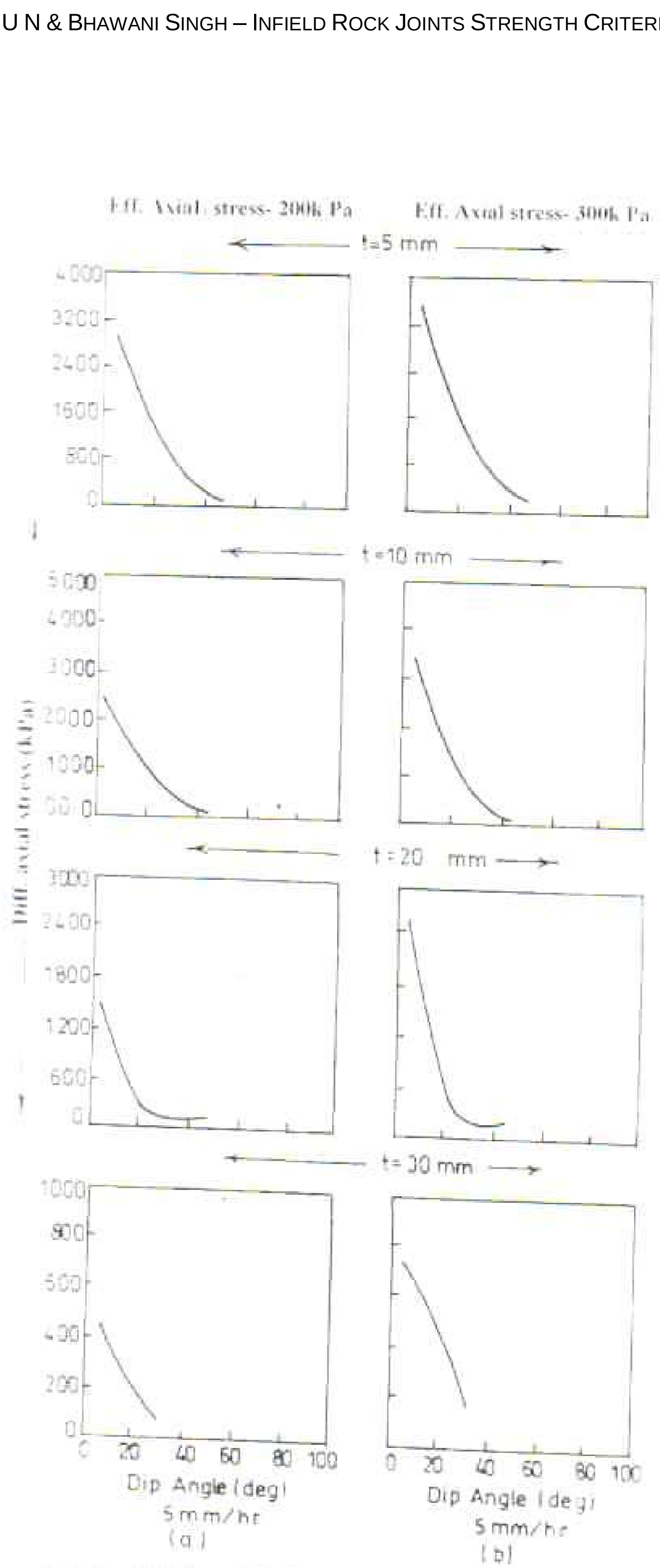

Fig. 11(a-b) Differential axial stress Vs Dip angle (Planar joint)

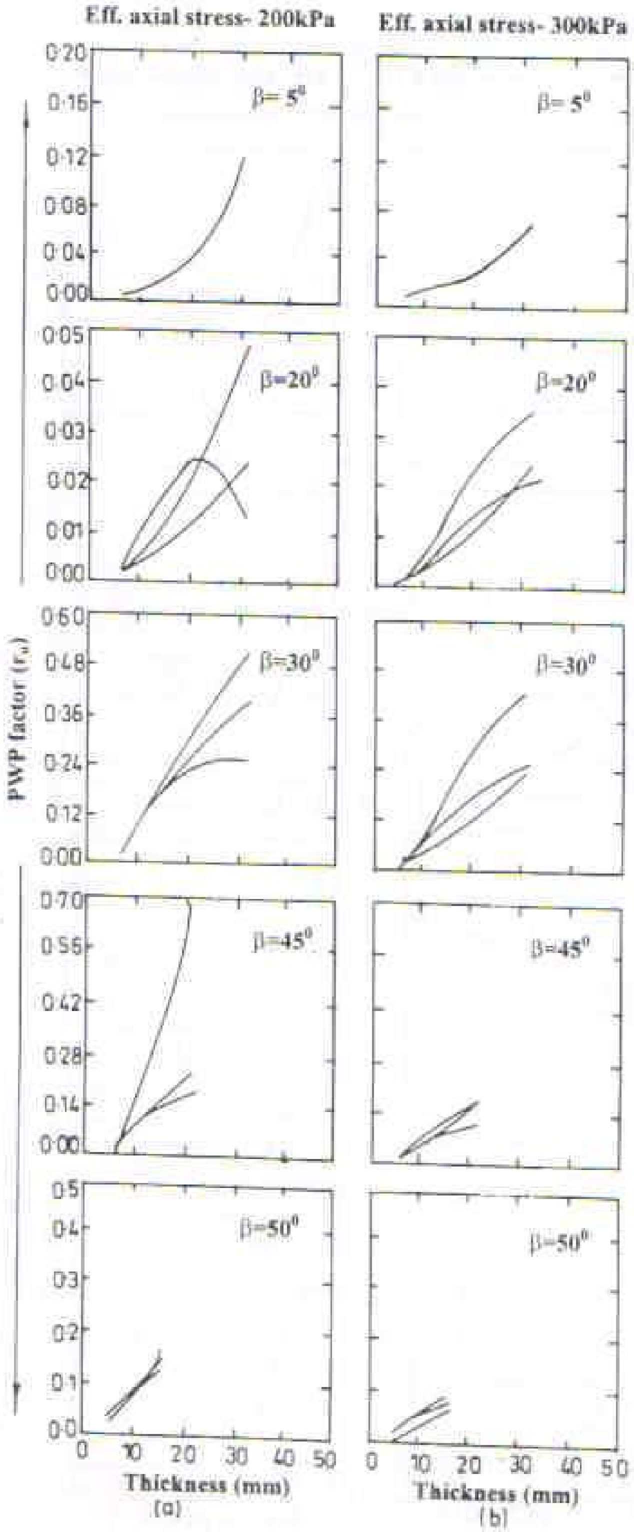

Fig.12(a-b) Pore water pressure factor(r_u) Vs Thickness (Undulating joint)

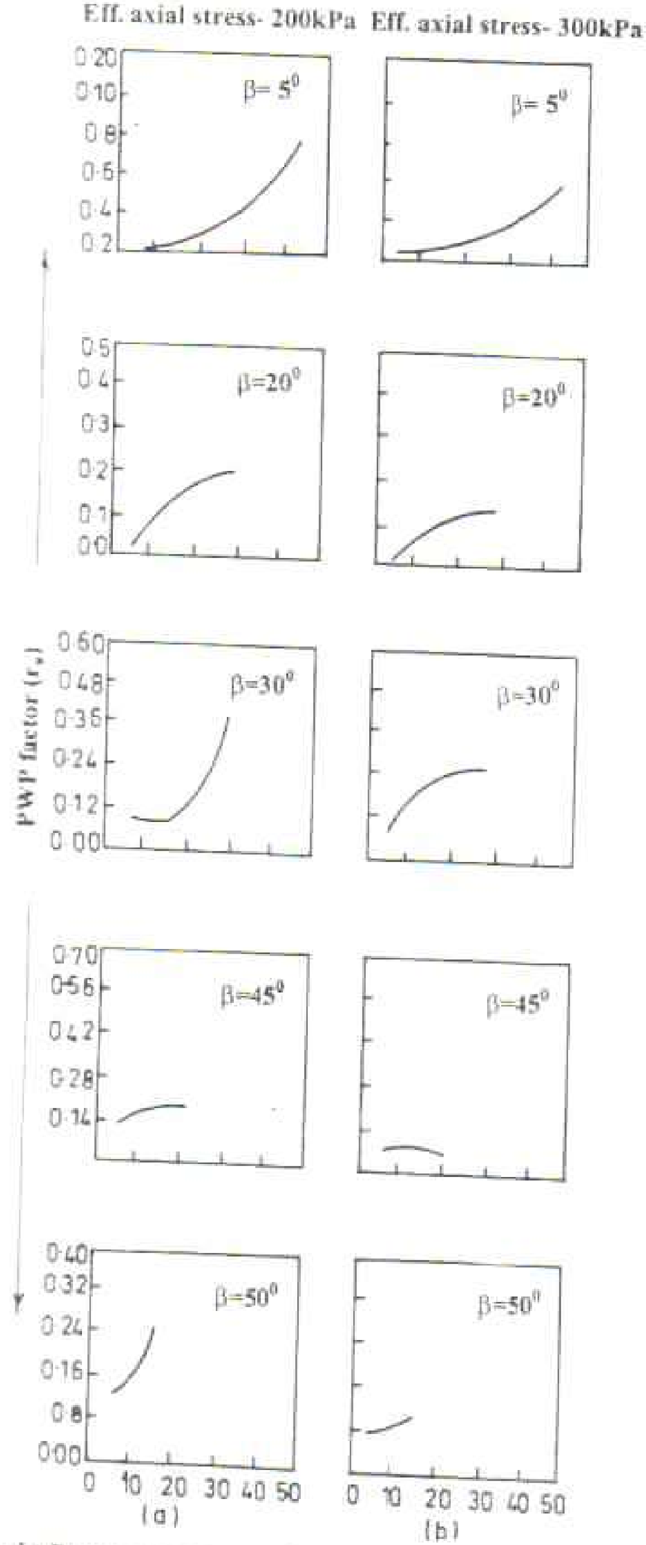

Fig. 13. (a-b) Pore water pressure factor(r_u) Vs Thickness (Planar joint)

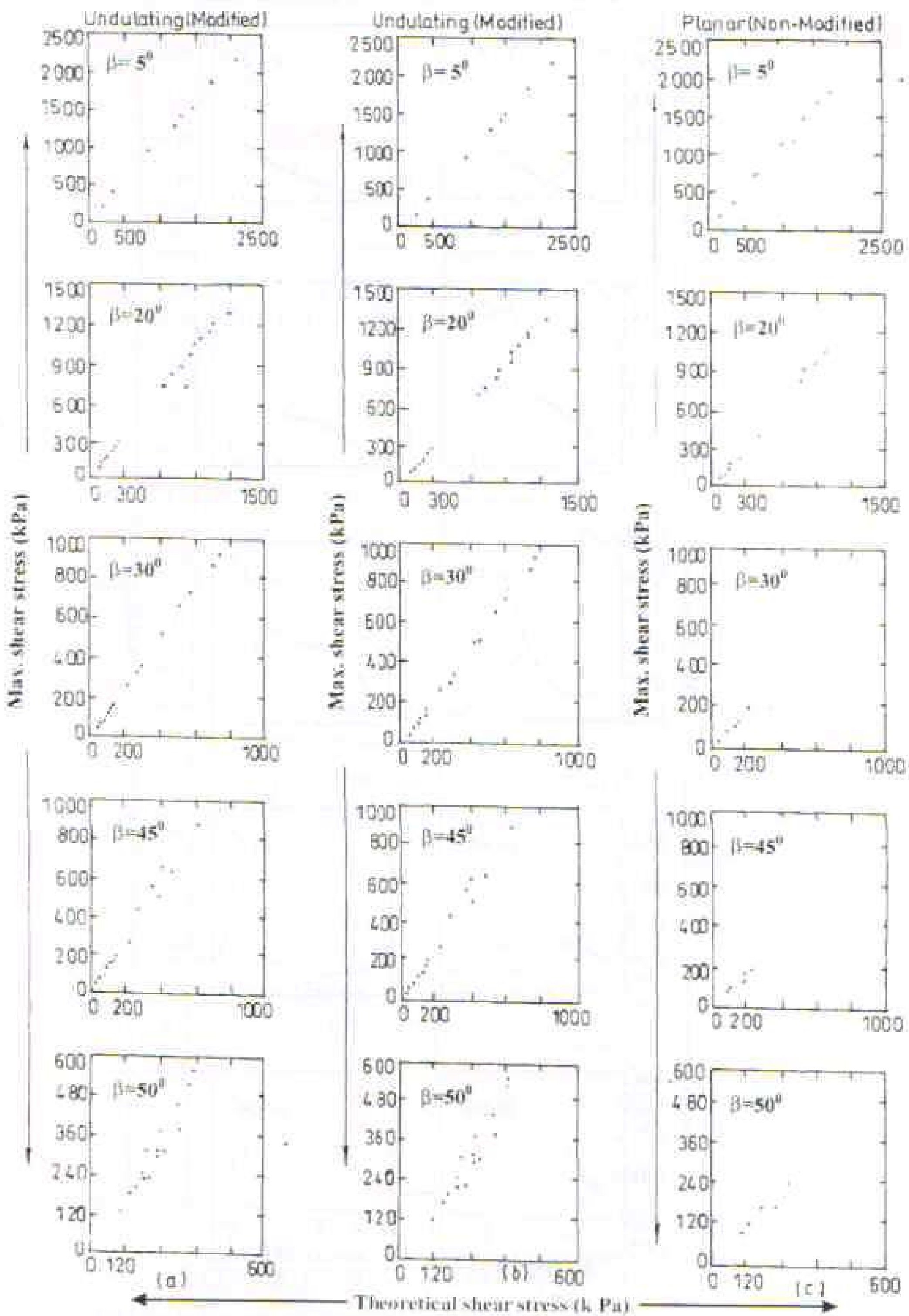

Fig. 14. (A-b) Relationship between experimental and theoretical strength (Based on Barton equation 1974)

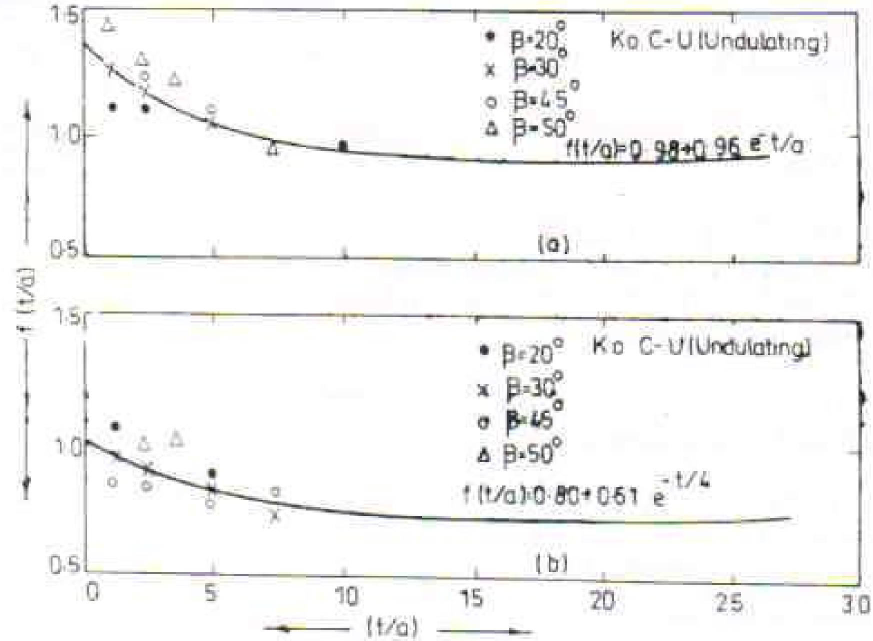

Fig. 15(a-b) Relationship between f(t/a) and t/a

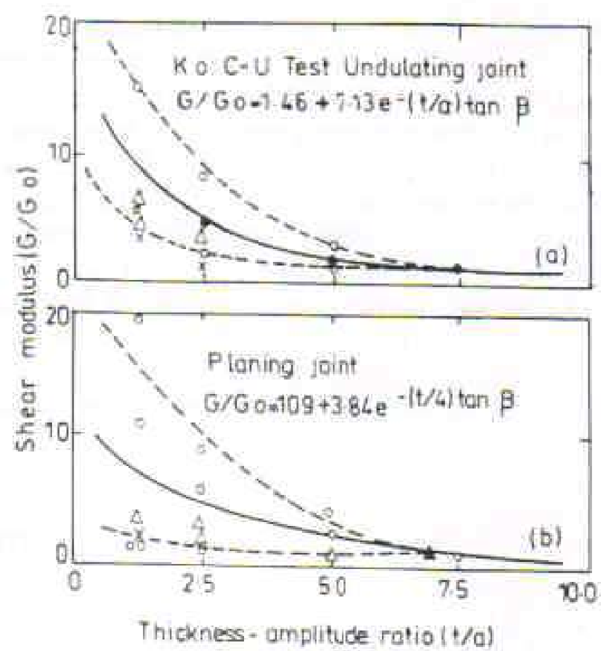

Fig. 16(a-b) Relationship between shear modulus (G/G_o) and t/a